SECTION

4.2

MINOR DRAINAGE SYSTEM DESIGN

4.2.1 Overview

4.2.1.1 Introduction

Minor stormwater drainage systems, also known as convenience systems, quickly remove runoff from areas such as streets and sidewalks for public safety purposes. The minor drainage system consists of inlets, street and roadway gutters, roadside ditches, small channels and swales, and small underground pipe systems which collect stormwater runoff and transport it to structural control facilities, pervious areas and/or the major drainage system (i.e., natural waterways, large man-made conduits, and large water impoundments).

This section is intended to provide criteria and guidance for the design of minor drainage system components including:

- Street and roadway gutters
- Stormwater inlets
- Storm drain pipe systems

Ditch, channel and swale design criteria and guidance are covered in Section 4.4, *Open Channel Design*.

Procedures for performing gutter flow calculations are based on a modification of Manning's Equation. Inlet capacity calculations for grate, curb and combination inlets are based on information contained in HEC-12 (USDOT, FHWA, 1984). Storm drain system design is based on the use of the Rational Formula. Alternate calculations may be used with the approval of Columbia County.

4.2.1.2 General Criteria

Design Frequency

See Section 4.1 for design storm requirements for the sizing of minor storm drainage system components.

Flow Spread Limits

Catch basins shall be spaced so that the spread in the street for the 25-year design flow shall not exceed the following, as measured from the face of the curb:

- 8 feet if the street is classified as a Collector or Arterial street (for 2-lane streets spread may extend to one-half of the travel lane; for 4-lane streets spread may extend across one travel lane)
- 16 feet at any given section, but in no case greater than 10 feet on one side of the street, if the street is classified as a Local or Sub-Collector street

4.2.2 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 4.2-1 will be used. These symbols were selected because of their wide use. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

Table 4.2-1 Symbols and Definitions

Symbol	Definition	Units
a	Gutter depression	in
A	Area of cross section	ft^2
d or D	Depth of gutter flow at the curb line	ft
D	Diameter of Pipe	ft
E_o	Ratio of frontal flow to total gutter flow Q_w/Q	--
g	Acceleration due to gravity (32.2 ft / s^2)	ft / s^2
h	Height of curb opening inlet	ft
H	Head loss	ft
K	Loss coefficient	--
L or L_T	Length of curb opening inlet	ft
L	Pipe length	ft
n	Manning's formula for triangular gutter flow	--
P	and side against curb	ft
Q	Rate of discharge in gutter	cfs
Q_i	Intercepted flow	cfs
Q_s	Gutter capacity above the depressed section	cfs
S or S_x	Cross Slope - Traverse slope	ft / ft
S or S_L	Longitudinal slope of pavement	ft / ft
S_f	Friction slope	ft / ft
S'_w	Depression section slope	ft / ft
T	Top width of water surface (spread on pavement)	ft
T_s	Spread above depressed section	ft
V	Velocity of flow	ft / s
W	Width of depression	ft
Z	T/d, reciprocal of the cross slope	--

4.2.3 Street and Roadway Gutters

Effective drainage of street and roadway pavements is essential to the maintenance of the roadway service level and to traffic safety. Water on the pavement can interrupt traffic flow, reduce skid resistance, increase potential for hydroplaning, limit visibility due to splash and spray, and cause difficulty in steering a vehicle when the front wheels encounter puddles. Surface drainage is a function of transverse and longitudinal pavement slope, pavement roughness, inlet spacing, and inlet capacity. The design of these elements is dependent on storm frequency and the allowable spread of stormwater on the pavement surface.

This section presents design guidance for gutter flow hydraulics originally published in HEC-12, Drainage of Highway Pavements and AASHTO's Model Drainage Manual.

4.2.3.1 Formula

The following form of Manning's Equation should be used to evaluate gutter flow hydraulics:

$$Q = [0.56 / n]\, S_x^{5/3}\, S^{1/2}\, T^{8/3} \qquad (4.2.1)$$

Where:
- Q = gutter flow rate, cfs
- n = Manning's roughness coefficient
- T = width of flow or spread, ft
- S_x = pavement cross slope, ft/ft
- S = longitudinal slope, ft/ft

4.2.3.2 Nomograph

Figure 4.2-1 is a nomograph for solving Equation 4.2.1. Manning's n values for various pavement surfaces are presented in Table 4.2-2 below.

4.2.3.3 Manning's n Table

Table 4.2-2 Manning's n Values for Street and Pavement Gutters

Type of Gutter or Pavement	Range of Manning's n
Concrete gutter, troweled finish	0.012
Asphalt pavement:	
Smooth texture	0.013
Rough texture	0.016
Concrete gutter with asphalt pavement:	
Smooth	0.013
Rough	0.015
Concrete pavement:	
Float finish	0.014
Broom finish	0.016
For gutters with small slopes, where sediment may accumulate, increase above values of n by	0.002

Note: Estimates are by the Federal Highw ay Administration

Source: USDOT, FHWA, HDS-3 (1961).

4.2.3.4 Uniform Cross Slope

The nomograph in Figure 4.2-1 is used with the following procedures to find gutter capacity for uniform cross slopes:

Condition 1: Find spread, given gutter flow.

Step 1: Determine input parameters, including longitudinal slope (S), cross slope (S_x), gutter flow (Q), and Manning's n.

Step 2: Draw a line between the S and S_x scales and note where it intersects the turning line.

Step 3: Draw a line between the intersection point from Step 2 and the appropriate gutter flow value on the capacity scale. If Manning's n is 0.016, use Q from Step 1; if not, use the product of Q and n.

Step 4: Read the value of the spread (T) at the intersection of the line from Step 3 and the spread scale.

Condition 2: Find gutter flow, given spread.

Step 1: Determine input parameters, including longitudinal slope (S), cross slope (S_x), spread (T), and Manning's n.

Step 2: Draw a line between the S and S_x scales and note where it intersects the turning line.

Step 3: Draw a line between the intersection point from Step 2 and the appropriate value on the T scale. Read the value of Q or Qn from the intersection of that line on the capacity scale.

Step 4: For Manning's n values of 0.016, the gutter capacity (Q) from Step 3 is selected. For other Manning's n values, the gutter capacity times n (Qn) is selected from Step 3 and divided by the appropriate n value to give the gutter capacity.

4.2.3.5 Composite Gutter Sections

Figure 4.2-2 in combination with Figure 4.2-1 can be used to find the flow in a gutter with width (W) less than the total spread (T). Such calculations are generally used for evaluating composite gutter sections or frontal flow for grate inlets.

Figure 4.2-3 provides a direct solution of gutter flow in a composite gutter section. The flow rate at a given spread or the spread at a known flow rate can be found from this figure. Figure 4.2-3 involves a complex graphical solution of the equation for flow in a composite gutter section. Typical of graphical solutions, extreme care in using the figure is necessary to obtain accurate results.

Condition 1: Find spread, given gutter flow.

Step 1: Determine input parameters, including longitudinal slope (S), cross slope (S_x), depressed section slope (S_w), depressed section width (W), Manning's n, gutter flow (Q), and a trial value of gutter capacity above the depressed section (Q_s).

Step 2: Calculate the gutter flow in W (Q_w), using the equation: $\mathbf{Q_w = Q - Q_s}$ **(4.2.2)**

Step 3: Calculate the ratios Q_w/Q or E_o and S_w/S_x and use Figure 4.2-2 to find an appropriate value of W/T.

Step 4: Calculate the spread (T) by dividing the depressed section width (W) by the value of W/T from Step 3.

Step 5: Find the spread above the depressed section (T_s) by subtracting W from the value of T obtained in Step 4.

Step 6: Use the value of T_s from Step 5 along with Manning's n, S, and S_x to find the actual value of Q_s from Figure 4.2-1.

Step 7: Compare the value of Q_s from Step 6 to the trial value from Step 1. If values are not comparable, select a new value of Q_s and return to Step 1.

Condition 2: Find gutter flow, given spread.

Step 1: Determine input parameters, including spread (T), spread above the depressed section (T_s), cross slope (S_x), longitudinal slope (S), depressed section slope (S_w), depressed section width (W), Manning's n, and depth of gutter flow (d).

Step 2: Use Figure 4.2-1 to determine the capacity of the gutter section above the depressed section (Q_s). Use the procedure for uniform cross slopes, substituting T_s for T.

Step 3: Calculate the ratios W/T and S_w/S_x, and, from Figure 4.2-2, find the appropriate value of E_o (the ratio of Q_w/Q).

Step 4: Calculate the total gutter flow using the equation:

$$\mathbf{Q = Q_s / (1 - E_o)} \qquad \textbf{(4.2.3)}$$

Where: Q = gutter flow rate, cfs

Q_s = flow capacity of the gutter section above the depressed section, cfs

E_o = ratio of frontal flow to total gutter flow (Q_w/Q)

Step 5: Calculate the gutter flow in width (W), using Equation 4.2.2.

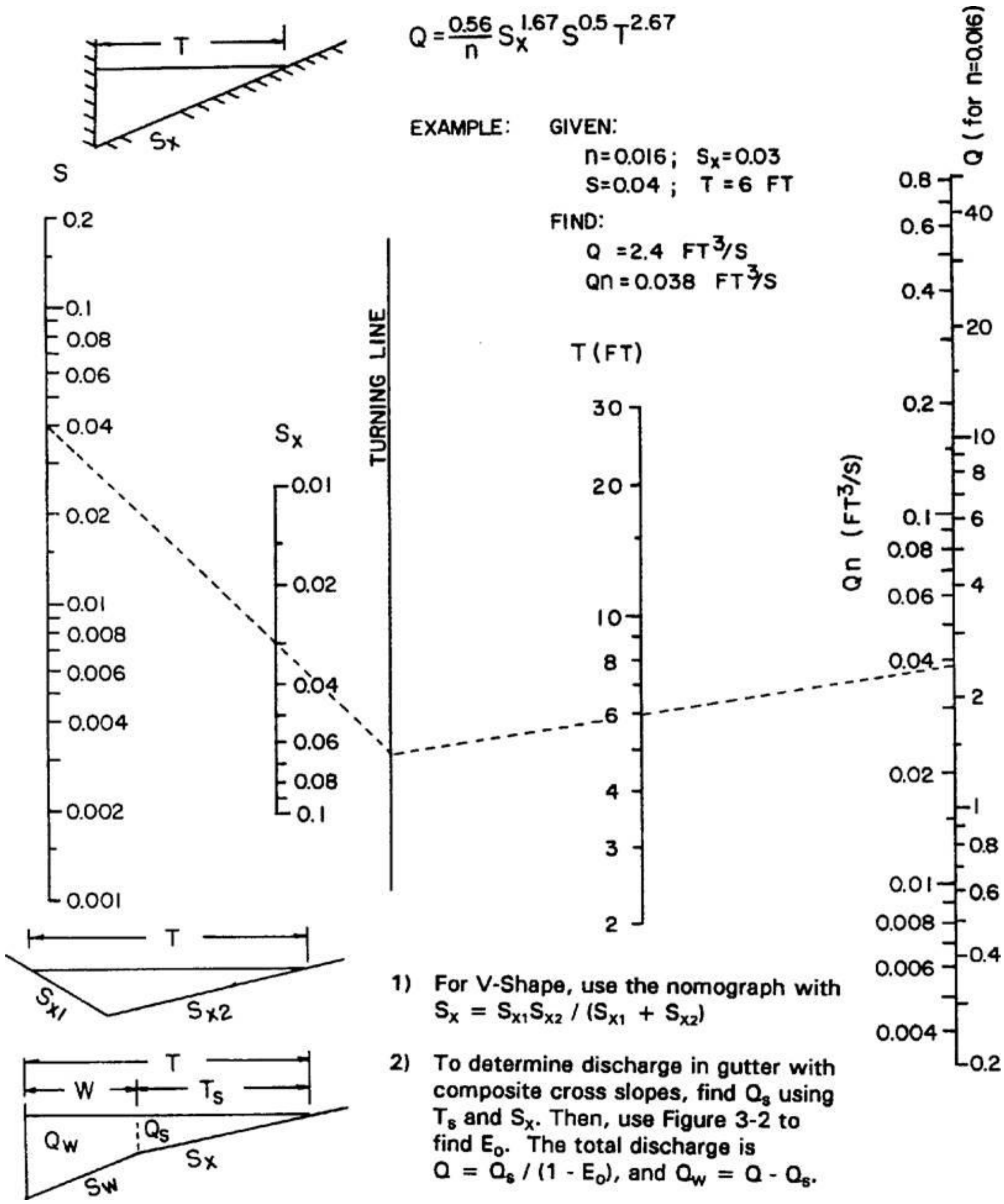


Figure 4.2-1 Flow in Triangular Gutter Sections

(Source: AASHTO Model Drainage Manual, 1991)

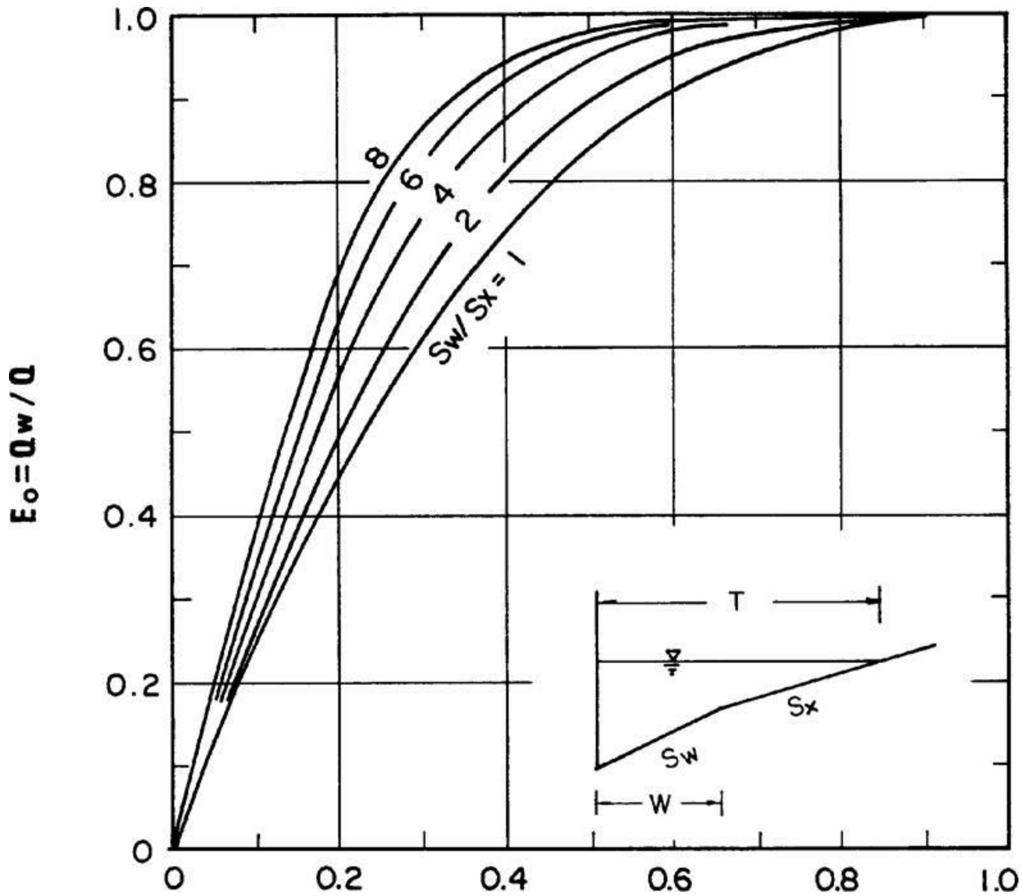


Figure 4.2-2 Ratio of Frontal Flow to Total Gutter Flow
(Source: AASHTO Model Drainage Manual, 1991)

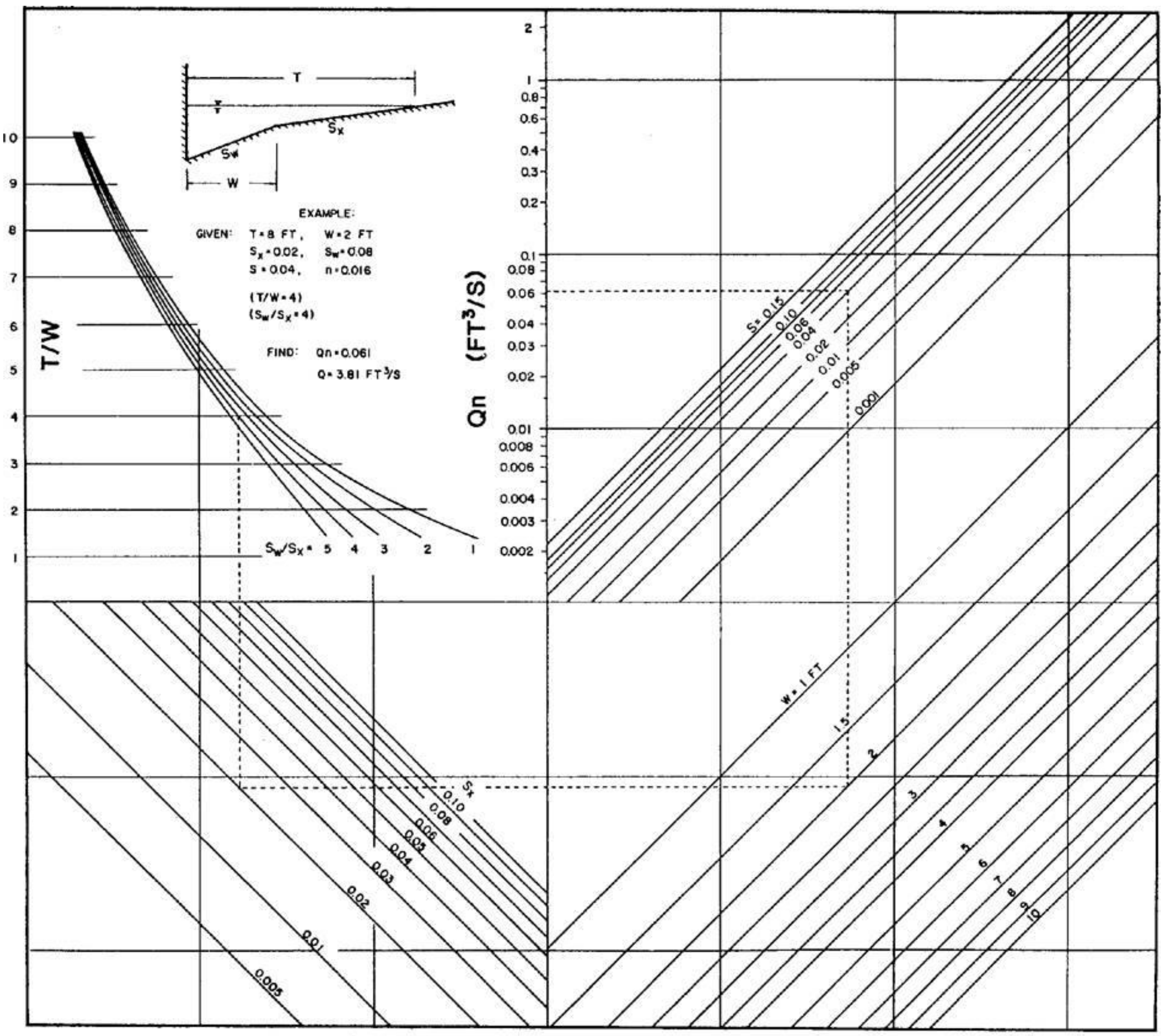


Figure 4.2-3 Flow in Composite Gutter Sections

(Source: AASHTO Model Drainage Manual, 1991)

4.2.3.6 Examples

Example 1

Given:	T	=	8 ft	S_x	=	0.025 ft/ft
	n	=	0.015	S	=	0.01 ft/ft

Find: (a) Flow in gutter at design spread

(b) Flow in width (W = 2 ft) adjacent to the curb

Solution: (a) From Figure 4.2-1, Q_n = 0.03

$$\mathbf{Q = Qn / n = 0.03 / 0.015 = 2.0\ cfs}$$

(b) T = 8 - 2 = 6 ft

$\mathbf{(Qn)_2 = 0.014}$ (Figure 4.2-1) (flow in 6-foot width outside of width (W))

$$\mathbf{Q = 0.014 / 0.015 = 0.9\ cfs}$$

$$\mathbf{Q_w = 2.0 - 0.9 = 1.1\ cfs}$$

Flow in the first 2 ft adjacent to the curb is 1.1 cfs and 0.9 cfs in the remainder of the gutter.

Example 2

Given:	T	=	6 ft	S_w	=	0.0833 ft/ft
	T_s	=	6 - 1.5 = 4.5 ft	W	=	1.5 ft
	S_x	=	0.03 ft/ft	n	=	0.014
	S	=	0.04 ft/ft			

Find: Flow in the composite gutter

Solution: (1) Use Figure 4.2-1 to find the gutter section capacity above the depressed section.

$$\mathbf{Q_s n = 0.038}$$

$$\mathbf{Q_s = 0.038 / 0.014 = 2.7\ cfs}$$

(2) Calculate W/T = 1.5/6 = 0.25 and

$$\mathbf{S_w / S_x = 0.0833 / 0.03 = 2.78}$$

Use Figure 4.2-2 to find E_o = 0.64

(3) Calculate the gutter flow using Equation 4.2.3

$$\mathbf{Q = 2.7 / (1 - 0.64) = 7.5\ cfs}$$

(4) Calculate the gutter flow in width, W, using Equation 4.2.2

$$\mathbf{Q_w = 7.5 - 2.7 = 4.8\ cfs}$$

4.2.4 Stormwater Inlets

Inlets are drainage structures used to collect surface water through grate or curb openings and convey it to storm drains or direct outlet to culverts. Grate inlets subject to traffic should be bicycle safe and be load-bearing adequate. Appropriate frames should be provided.

Inlets used for the drainage of highway surfaces can be divided into three major classes:

- Grate Inlets – These inlets include grate inlets consisting of an opening in the gutter covered by one or more grates, and slotted inlets consisting of a pipe cut along the longitudinal axis with a grate or spacer bars to form slot openings.
- Curb-Opening Inlets – These inlets are vertical openings in the curb covered by a top slab.

- Combination Inlets – These inlets usually consist of both a curb-opening inlet and a grate inlet placed in a side-by-side configuration, but the curb opening may be located in part upstream of the grate.

Inlets may be classified as being on a continuous grade or in a sump. The term "continuous grade" refers to an inlet located on the street with a continuous slope past the inlet with water entering from one direction. The "sump" condition exists when the inlet is located at a low point and water enters from both directions.

Where significant ponding can occur, in locations such as underpasses and in sag vertical curves in depressed sections, it is good engineering practice to place flanking inlets on each side of the inlet at the low point in the sag. The flanking inlets should be placed so that they will limit spread on low gradient approaches to the level point and act in relief of the inlet at the low point if it should become clogged or if the design spread is exceeded.

The design of grate inlets will be discussed in subsection 4.2.5, curb inlet design in Section 4.2.6, and combination inlets in Section 4.2.7.

4.2.5 Grate Inlet Design

4.2.5.1 Grate Inlets on Grade

The capacity of an inlet depends upon its geometry and the cross slope, longitudinal slope, total gutter flow, depth of flow and pavement roughness. The depth of water next to the curb is the major factor in the interception capacity of both gutter inlets and curb opening inlets. At low velocities, all of the water flowing in the section of gutter occupied by the grate, called frontal flow, is intercepted by grate inlets, and a small portion of the flow along the length of the grate, termed side flow, is intercepted. On steep slopes, only a portion of the frontal flow will be intercepted if the velocity is high or the grate is short and splash-over occurs. For grates less than 2 feet long, intercepted flow is small.

A parallel bar grate is the most efficient type of gutter inlet; however, when crossbars are added for bicycle safety, the efficiency is greatly reduced. Where bicycle traffic is a design consideration, the curved vane grate and the tilt bar grate are recommended for both their hydraulic capacity and bicycle safety features. They also handle debris better than other grate inlets but the vanes of the grate must be turned in the proper direction. Where debris is a problem, consideration should be given to debris handling efficiency rankings of grate inlets from laboratory tests in which an attempt was made to qualitatively simulate field conditions. Table 4.2-3 presents the results of debris handling efficiencies of several grates.

The ratio of frontal flow to total gutter flow, E_o, for straight cross slope is expressed by the following equation:

$$\mathbf{E_o = Q_w / Q = 1 - (1 - W / T)^{2.67}} \tag{4.2.4}$$

Where: Q = total gutter flow, cfs

Q_w = flow in width W, cfs

W = width of depressed gutter or grate, ft

T = total spread of water in the gutter, ft

Figure 4.2-2 provides a graphical solution of E_o for either depressed gutter sections or straight cross slopes. The ratio of side flow, Q_s, to total gutter flow is:

$$\mathbf{Q_s / Q = 1 - Q_w / Q = 1 - E_o} \tag{4.2.5}$$

Table 4.2-3 Grate Debris Handling Efficiencies

Rank	Grate	Longitudinal Slope (0.005)	Longitudinal Slope (0.04)
1	CV - 3 1/4 - 4 1/4	46	61
2	30 - 3 1/4 - 4	44	55
3	45 - 3 1/4 - 4	43	48
4	P - 1 7/8	32	32
5	P - 1 7/8 - 4	18	28
6	45 - 2 1/4 - 4	16	23
7	Recticuline	12	16
8	P - 1 1/18	9	20

Source: 'Drainage of Highway Pavements" (HEC-12), Federal Highway Administration, 1984.

The ratio of frontal flow intercepted to total frontal flow, R_f, is expressed by the following equation:

$$R_f = 1 - 0.09(V - V_o) \qquad \textbf{(4.2.6)}$$

Where: V = velocity of flow in the gutter, ft/s (using Q from Figure 4.2-1)

V_o = gutter velocity where splash-over first occurs, ft/s (from Figure 4.2-4)

This ratio is equivalent to frontal flow interception efficiency. Figure 4.2-4 provides a solution of equation 4.2.6, which takes into account grate length, bar configuration and gutter velocity at which splash-over occurs. The gutter velocity needed to use Figure 4.2-4 is total gutter flow divided by the area of flow. The ratio of side flow intercepted to total side flow, R_s, or side flow interception efficiency, is expressed by:

$$R_s = 1 / [1 + (0.15V^{1.8} / S_x L^{2.3})] \qquad \textbf{(4.2.7)}$$

Where: L = length of the grate, ft

Figure 4.2-5 provides a solution to equation 4.2.7.

The efficiency, E, of a grate is expressed as:

$$E = R_f E_o + R_s \ (1 - E_o)] \qquad \textbf{(4.2.8)}$$

The interception capacity of a grate inlet on grade is equal to the efficiency of the grate multiplied by the total gutter flow:

$$Q_i = EQ = Q\ [R_f E_o + R_s (1 - E_o)] \qquad \textbf{(4.2.9)}$$

The following example illustrates the use of this procedure.

Given: W = 2 ft   T = 8 ft

S_x = 0.025 ft/ft   S = 0.01 ft/ft

E_o = 0.69   Q = 3.0 cfs

V = 3.1 ft/s   Gutter depression = 2 in

Find: Interception capacity of:

(1) a curved vane grate, and

(2) a reticuline grate 2-ft long and 2-ft wide

Solution:

From Figure 4.2-4 for Curved Vane Grate, R_f = 1.0

From Figure 4.2-4 for Reticuline Grate, R_f = 1.0

From Figure 4.2-5 R_s = 0.1 for both grates

From Equation 4.2.9:

$$Q_i = 3.0\,[1.0 \times 0.69 + 0.1\,(1 - 0.69)] = 2.2 \text{ cfs}$$

For this example, the interception capacity of a curved vane grate is the same as that for a reticuline grate for the sited conditions.

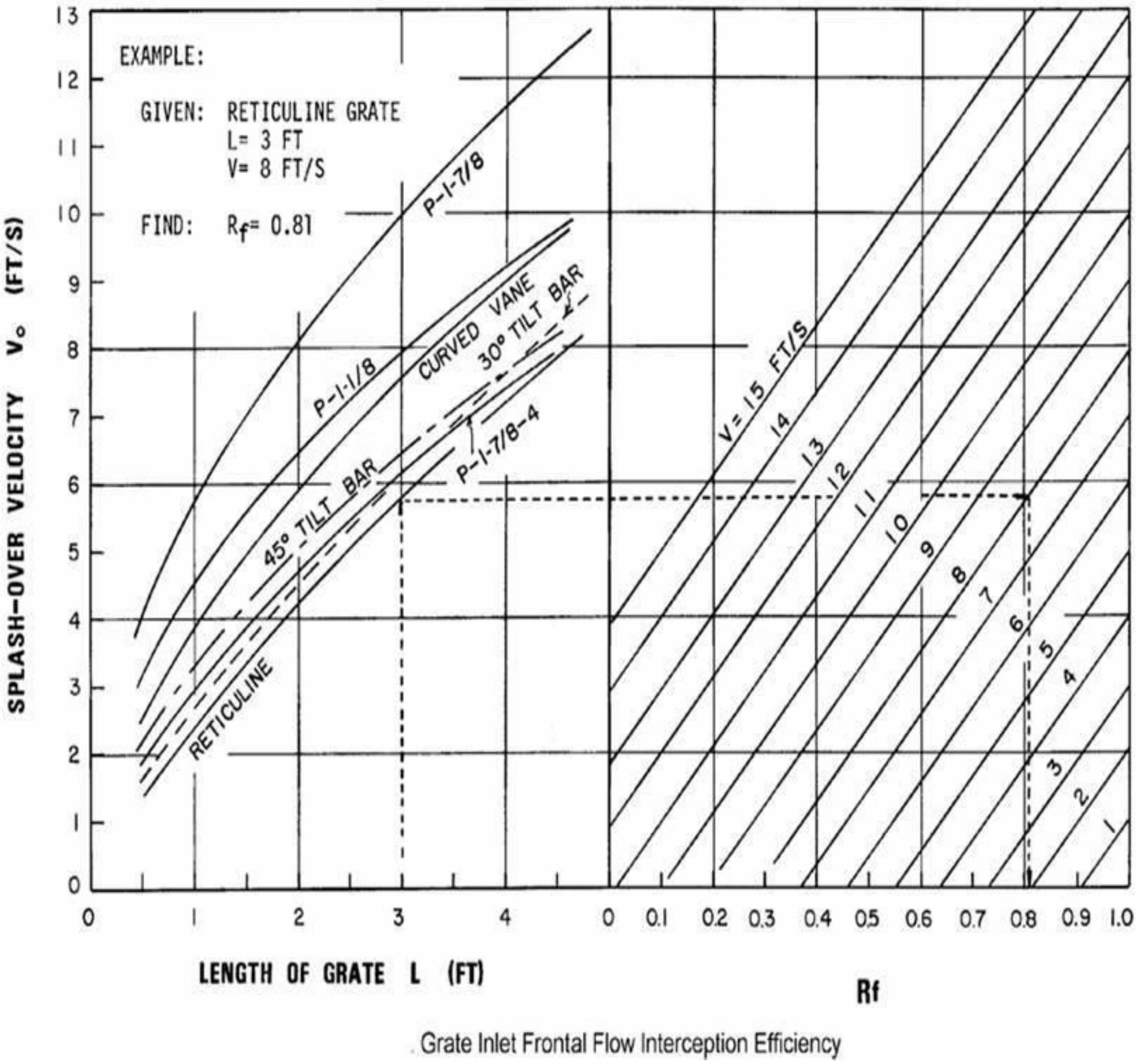


Figure 4.2-4 Grate Inlet Frontal Flow Interception Efficiency
(Source: HEC-12, 1984)

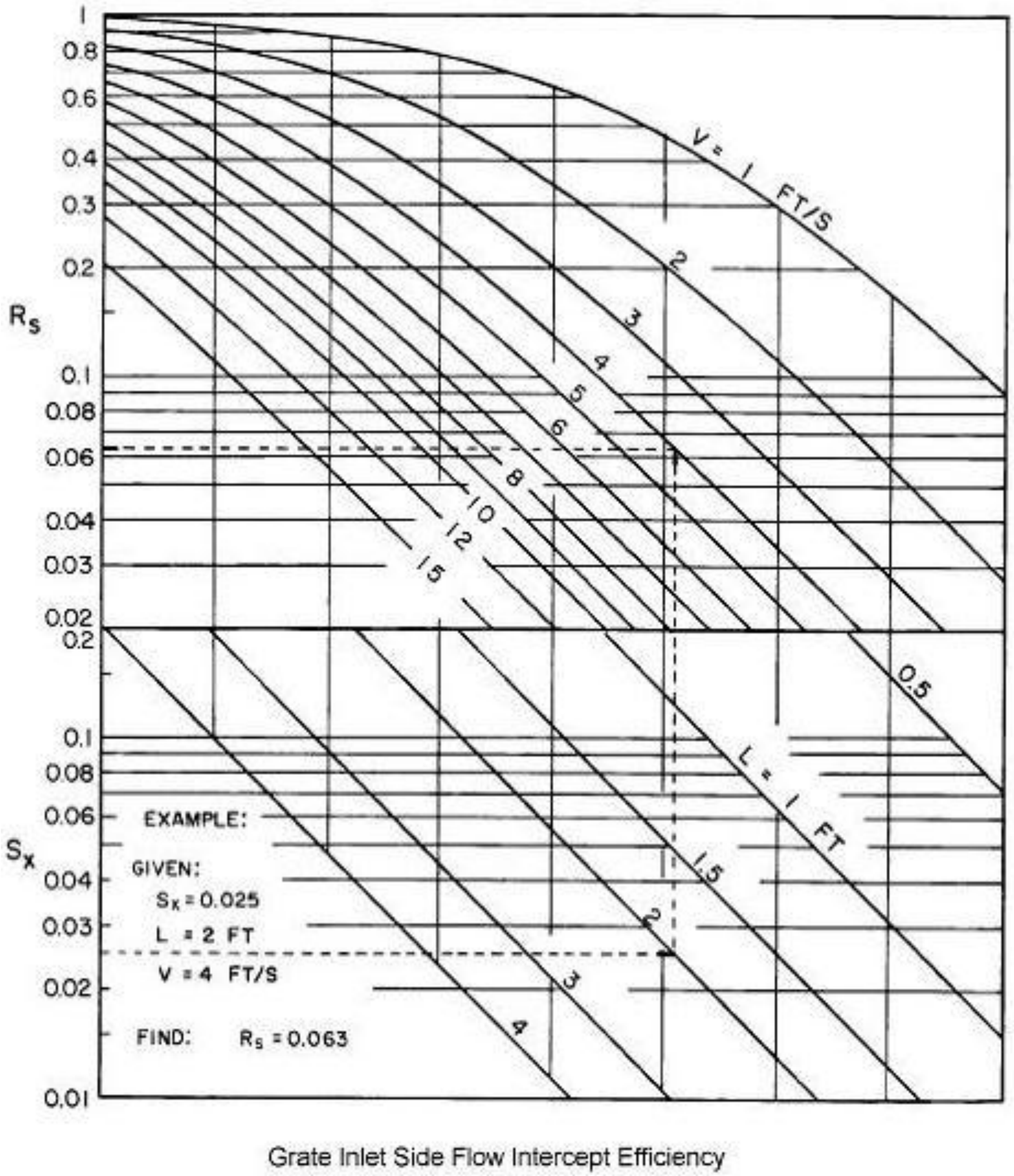


Figure 4.2-5 Grate Inlet Side Frontal Flow Interception Efficiency

(Source: HEC-12, 1984)

4.2.5.2 Grate Inlets in Sag

A grate inlet in a sag operates as a weir up to a certain depth, depending on the bar configuration and size of the grate, and as an orifice at greater depths. For a standard gutter inlet grate, weir operation continues to a depth of about 0.4 feet above the top of grate and when depth of water exceeds about 1.4 feet, the grate begins to operate as an orifice. Between depths of about 0.4 feet and about 1.4 feet, a transition from weir to orifice flow occurs.

The capacity of grate inlets operating as a weir is:

$$\mathbf{Q_i = CPd^{1.5}} \qquad \mathbf{(4.2.10)}$$

Where: P = perimeter of grate excluding bar widths and the side against the curb, ft

C = 3.0

d = depth of water above grate, ft

and as an orifice is:

$$\mathbf{Q_i = CA\ (2gd)\ 0.5} \qquad \mathbf{(4.2.11)}$$

Where: C = 0.67 orifice coefficient

A = clear opening area of the grate, ft^2

g = 32.2 ft/s^2

Figure 4.2-6 is a plot of equations 4.2. 10 and 4.2.11 for various grate sizes. The effects of grate size on the depth at which a grate operates as an orifice, is apparent from the chart. Transition from weir to orifice flow results in interception capacity less than that computed by either weir or the orifice equation. This capacity can be approximated by drawing in a curve between the lines representing the perimeter and net area of the grate to be used. The following example illustrates the use of this figure.

Given: Symmetrical sag vertical curve with equal bypass from inlets upgrade of the low point; allow for 50% clogging of the grate.

Q_b = 3.6 cfs Q = 8 cfs, 25-year storm

T = 10 ft, design S_x = 0.05 ft/ft d = TS_x = 0.5 ft

Find: Grate size for design Q. Check spread at S = 0.003 on approaches to the low point.

Solution: From Figure 4.2-6, a grate must have a perimeter of 8 ft to intercept 8 cfs at a depth of 0.5 ft.

Some assumptions must be made regarding the nature of the clogging in order to compute the capacity of a partially clogged grate. If the area of a grate is 50% covered by debris so that the debris-covered portion does not contribute to interception, the effective perimeter will be reduced by a lesser amount than 50%. For example if a 2-ft x 4-ft grate is clogged so that the effective width is 1 ft, then the perimeter, P = 1 + 4 + 1 = 6 ft, rather than 8 ft, the total perimeter, or 4 ft, half of the total perimeter. The area of the opening would be reduced by 50% and the perimeter by 25%.

Therefore, assuming 50% clogging along the length of the grate, a 4 x 4, a 2 x 6, or a 3 x 5 grate would meet requirements of an 8-ft perimeter 50% clogged.

Assuming that the installation chosen to meet design conditions is a double 2 x 3 ft grate, for 50% clogged conditions: P = 1 + 6 + 1 = 8 ft

For 25-year flow: d = 0.5 ft (from Figure 4.2-6)

The American Society of State Highway and Transportation Officials (AASHTO) geometric policy recommends a gradient of 0.3% within 50 ft of the level point in a sag vertical curve.

Check T at S = 0.003 for the design and check flow:

Q = 3.6 cfs, T = 8.2 ft (25-year storm) (from Figure 4.2-1)

Thus a double 2 x 3-ft grate 50% clogged is adequate to intercept the design flow at a spread that does not exceed design spread, and spread on the approaches to the low point will not exceed design spread. However, the tendency of grate inlets to clog completely warrants consideration of a combination inlet, or curb-opening inlet in sag where ponding can occur, and flanking inlets on the low gradient approaches.

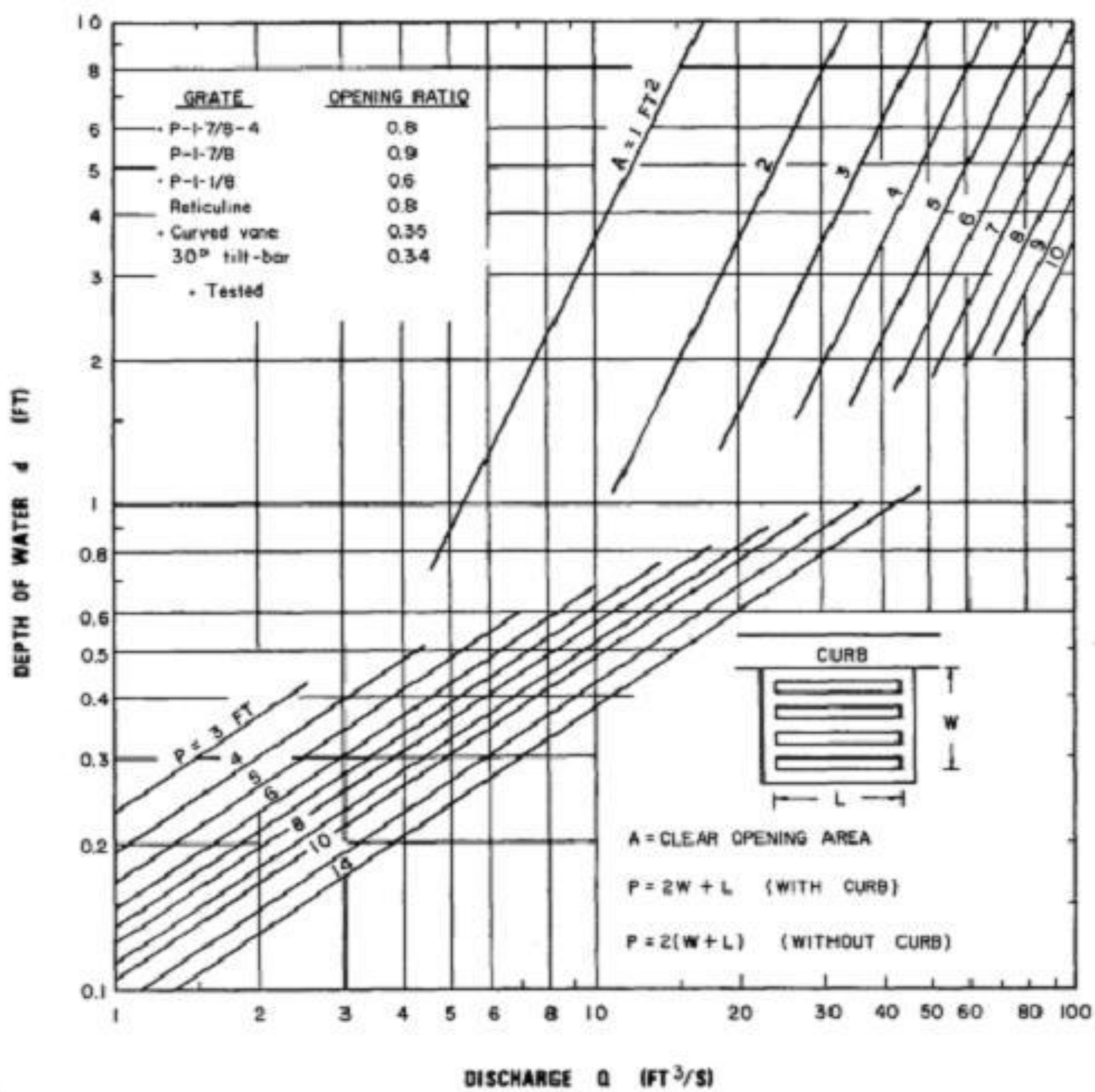


Figure 4.2-6 Grate Inlet Capacity in Sag Conditions
(Source: HEC-12, 1984)

4.2.6 Curb Inlet Design

4.2.6.1 Curb Inlets on Grade

Following is a discussion of the procedures for the design of curb inlets on grade. Curb-opening inlets are effective in the drainage of highway pavements where flow depth at the curb is sufficient for the inlet to perform efficiently. Curb openings are relatively free of clogging tendencies and offer little interference to traffic operation. They are a viable alternative to grates in many locations where grates would be in traffic lanes or would be hazardous for pedestrians or bicyclists.

The length of curb-opening inlet required for total interception of gutter flow on a pavement section with a straight cross slope is determined using Figure 4.2-7. The efficiency of curb- opening inlets shorter than the length required for total interception is determined using Figure 4.2-8.

The length of inlet required for total interception by depressed curb-opening inlets or curb-openings in depressed gutter sections can be found by the use of an equivalent cross slope, S_e, in the following equation:

$$S_e = S_x + S'_w E_o \qquad \textbf{(4.2.12)}$$

Where: E_o = ratio of flow in the depressed section to total gutter flow

S'_w = cross slope of gutter measured from the cross slope of the pavement, S_x

S'_w = (a/12W)

Where: a = gutter depression, in

w = width of depressed gutter, ft

It is apparent from examination of Figure 4.2-7 that the length of curb opening required for total interception can be significantly reduced by increasing the cross slope or the equivalent cross slope. The equivalent cross slope can be increased by use of a continuously depressed gutter section or a locally depressed gutter section.

Design Steps

Steps for using Figures 4.2-7 and 4.2-8 in the design of curb inlets on grade are given below.

Step 1: Determine the following input parameters:

Cross slope =	S_x (ft/ft)	Longitudinal slope =	S (ft/ft)
Gutter flow rate =	Q (cfs)	Manning's n =	n

Spread of water on pavement = T (ft) from Figure 4.2-1

Step 2: Enter Figure 4.2-7 using the two vertical lines on the left side labeled n and S. Locate the value for Manning's n and longitudinal slope and draw a line connecting these points and extend this line to the first turning line.

Step 3: Locate the value for the cross slope (or equivalent cross slope) and draw a line from the point on the first turning line through the cross slope value and extend this line to the second turning line.

Step 4: Using the far right vertical line labeled Q locate the gutter flow rate. Draw a line from this value to the point on the second turning line. Read the length required from the vertical line labeled L_T.

Step 5: If the curb-opening inlet is shorter than the value obtained in Step 4, Figure 4.2-8 can be used to calculate the efficiency. Enter the x-axis with the L/L_T ratio and draw a vertical line upward to the E curve. From the point of intersection, draw a line horizontally to the intersection with the y-axis and read the efficiency value.

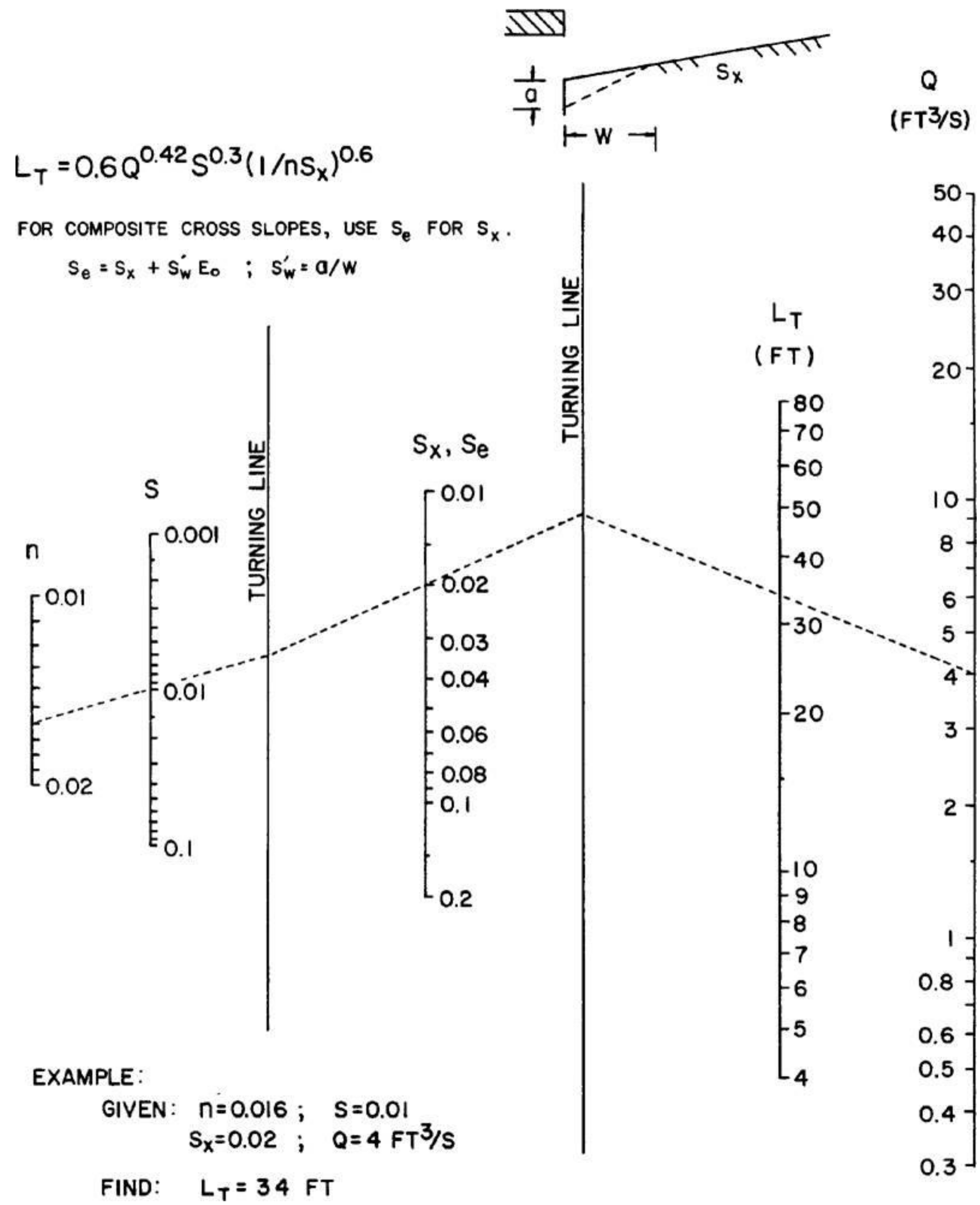


Figure 4.2-7 Curb-Opening and Slotted Drain Inlet Length for Total Interception
(Source: HEC-12, 1984)

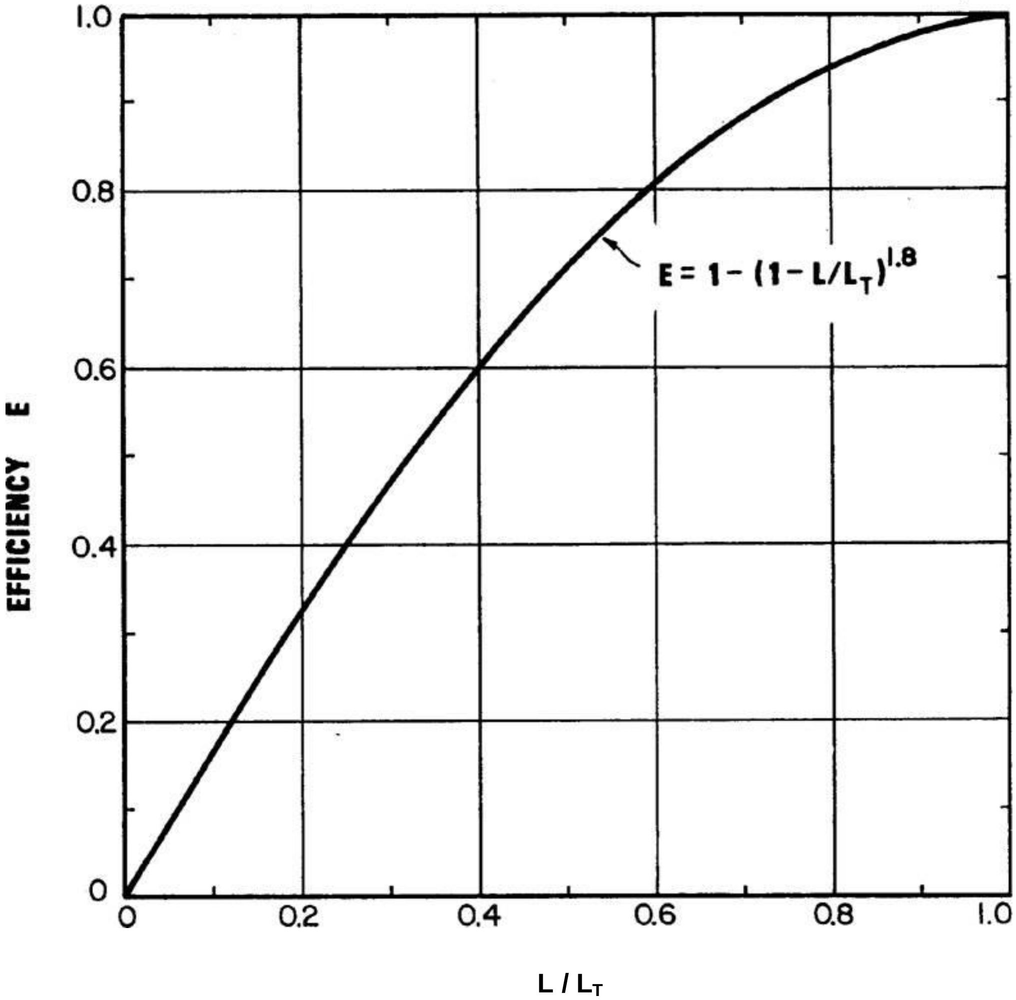


Figure 4.2-8 Curb-Opening and Slotted Drain Inlet Interception Efficiency
(Source: HEC-12, 1984)

Example

Given: S_x = 0.03 ft/ft   n = 0.016

S = 0.035 ft/ft   Q = 5 cfs

S'_w = 0.083 (a = 2 in, W = 2 ft)

Find: (1) Q_i for a 10-ft curb-opening inlet

(2) Q_i for a depressed 10-ft curb-opening inlet with a = 2 in, W = 2 ft, T = 8 ft (Figure 4.2-1)

Solution: (1) From Figure 4.2-7, $\mathbf{L_T = 41\ ft, L/L_T = 10/41 = 0.24}$

From Figure 4.2-8, $\mathbf{E = 0.39, Q_i = EQ = 0.39 * 5 = 2\ cfs}$

(2) $\mathbf{Qn = 5.0 * 0.016 = 0.08\ cfs}$

$\mathbf{S_w / S_x = (0.03 + 0.083) / 0.03 = 3.77}$

$\mathbf{T / W = 3.5}$ (from Figure 4.2-3) $\mathbf{T = 3.5 * 2 = 7\ ft}$

$\mathbf{W / T = 2/7 = 0.29\ ft}$

$\mathbf{E_o = 0.72}$ (from Figure 4.2-2)

Therefore, $\mathbf{S_e = S_x + S'_w E_o = 0.03 + 0.083\,(0.72) = 0.09}$

From Figure 4.2-7, $\mathbf{L_T = 23\ ft, L/L_T = 10/23 = 0.4}$

From Figure 4.2-8, $\mathbf{E = 0.64, Q_i = 0.64 * 5 = 3.2\ cfs}$

The depressed curb-opening inlet will intercept 1.6 times the flow intercepted by the undepressed curb opening and over 60% of the total flow.

4.2.6.2 Curb Inlets in Sump

For the design of a curb-opening inlet in a sump location, the inlet operates as a weir to depths equal to the curb opening height and as an orifice at depths greater than 1.4 times the opening height. At depths between 1.0 and 1.4 times the opening height, flow is in a transition stage.

The capacity of curb-opening inlets in a sump location can be determined from Figure 4.2-9, which accounts for the operation of the inlet as a weir and as an orifice at depths greater than 1.4h. This figure is applicable to depressed curb-opening inlets and the depth at the inlet includes any gutter depression. The height (h) in the figure assumes a vertical orifice opening (see sketch on Figure 4.2-9). The weir portion of Figure 4.2-9 is valid for a depressed curb- opening inlet when $\mathbf{d \leq (h + a/12)}$.

The capacity of curb-opening inlets in a sump location with a vertical orifice opening but without any depression can be determined from Figure 4.2-10. The capacity of curb-opening inlets in a sump location with other than vertical orifice openings can be determined by using Figure 4.2-11.

Design Steps

Steps for using Figures 4.2-9, 4.2-10, and 4.2-11 in the design of curb-opening inlets in sump locations are given below.

Step 1: Determine the following input parameters:

Cross slope = S_x (ft/ft)

Spread of water on pavement = T (ft) from Figure 4.2-1

Gutter flow rate = Q (cfs) or dimensions of curb-opening inlet [L (ft) and H (in)]

Dimensions of depression if any [a (in) and W (ft)]

Step 2: To determine discharge given the other input parameters, select the appropriate figure (4.2-9, 4.2-10, or 4.2-11 depending on whether the inlet is in a depression and if the orifice opening is vertical).

Step 3: To determine the discharge (Q), given the water depth (d), locate the water depth value on the y-axis and draw a horizontal line to the appropriate perimeter (p), height (h), length (L), or width x length (hL) line. At this intersection draw a vertical line down to the x-axis and read the discharge value.

Step 4: To determine the water depth given the discharge, use the procedure described in Step 3 except enter the figure at the value for the discharge on the x-axis.

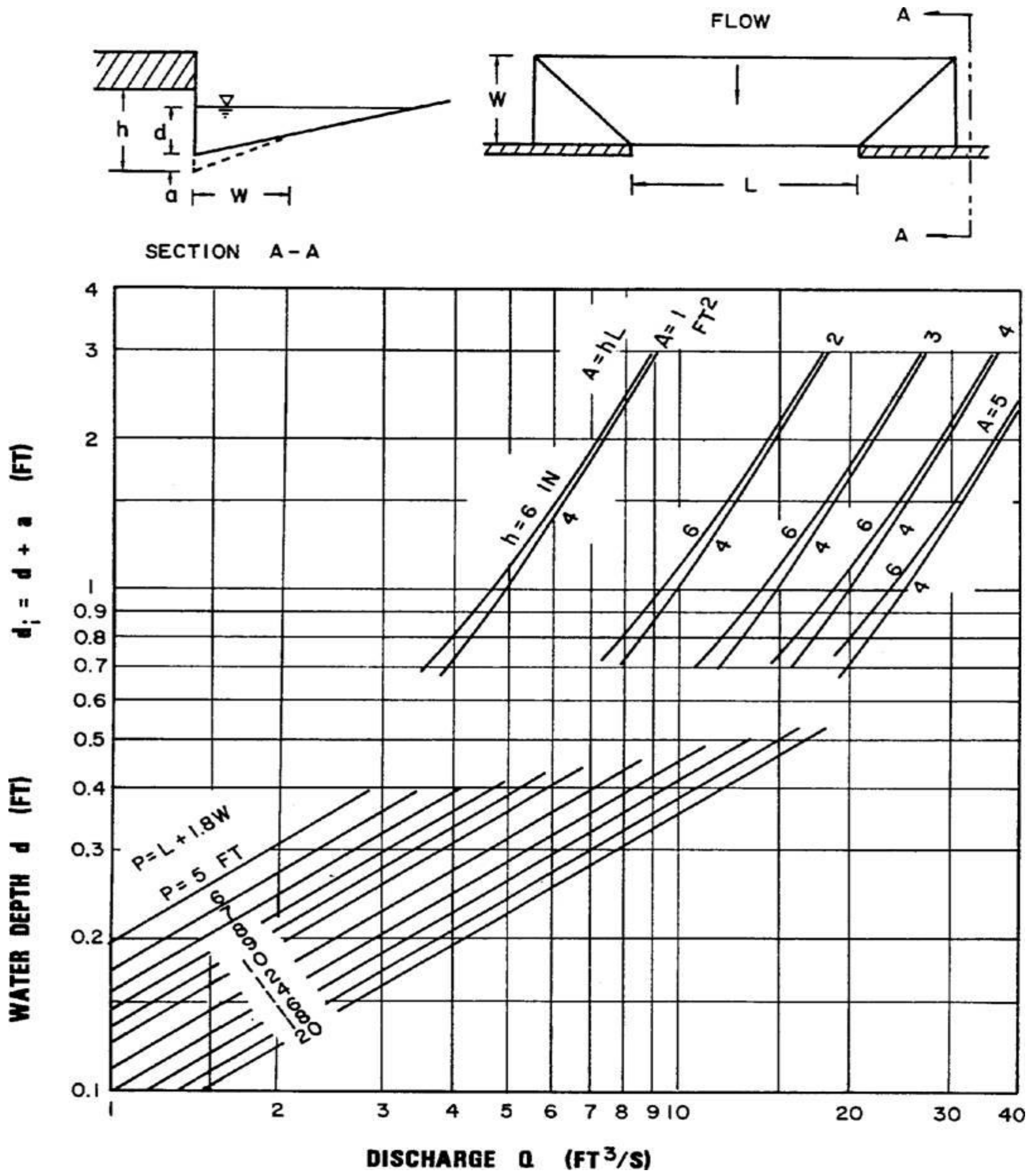


Figure 4.2-9 Depressed Curb-Opening Inlet Capacity in Sump Locations
(Source: AASHTO Model Drainage Manual, 1991)

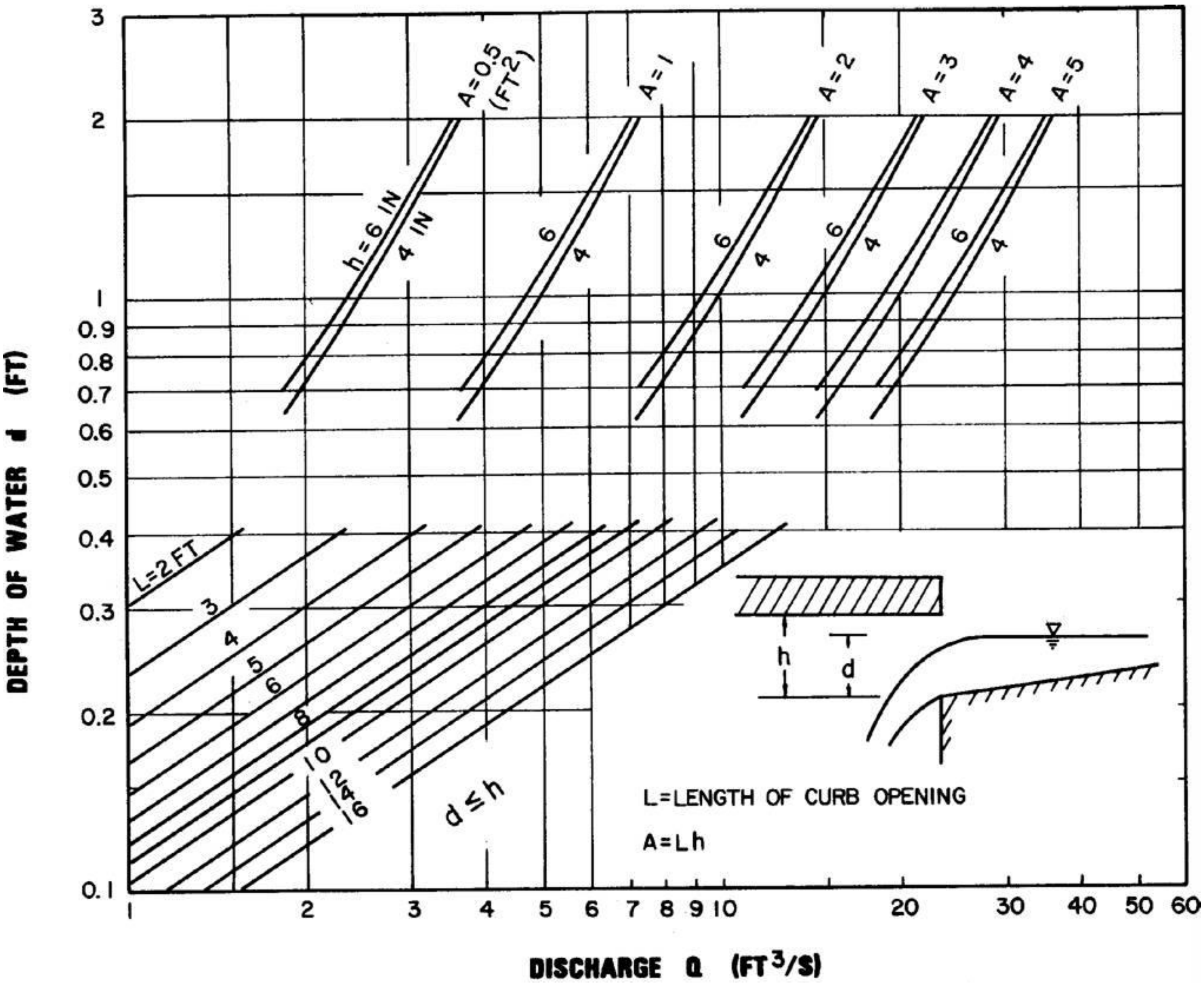


Figure 4.2-10 Curb-Opening Inlet Capacity in Sump Locations
(Source: AASHTO Model Drainage Manual, 1991)

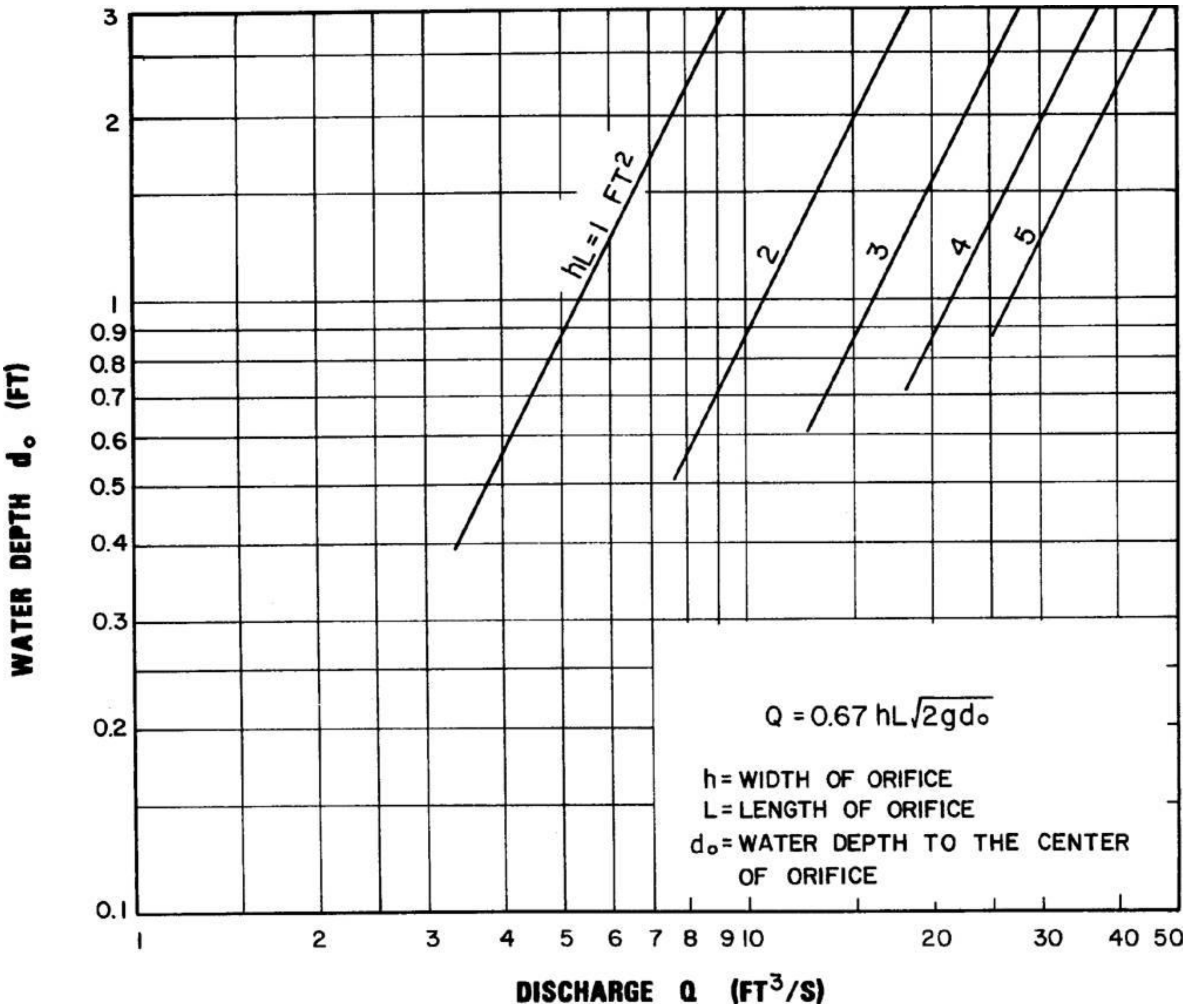


Figure 4.2-11 Curb-Opening Inlet Orifice Capacity for Inclined and Vertical Orifice Throats

(Source: AASHTO Model Drainage Manual, 1991)

Example

Given: Curb-opening inlet in a sump location

$L = 5$ ft
$h = 5$ in

(1) Undepressed curb opening

$S_x = 0.05$ ft/ft
$T = 8$ ft

(2) Depressed curb opening

$S_x = 0.05$ ft/ft
$a = 2$ in
$W = 2$ ft
$T = 8$ ft

Find: Discharge Q_i

Solution: (1) $\mathbf{d = TS_x = 8 * 0.05 = 0.4\ ft}$

$\mathbf{d < h}$

From Figure 4.2-10, Qi = 3.8 cfs

(2) $\mathbf{d = 0.4\ ft}$

$\mathbf{h = a/12 = (5 + 2/12)/12 = 0.43\ ft}$

since $\mathbf{d < 0.43}$ the weir portion of Figure 4.2-9 is applicable (lower portion of the figure).

$\mathbf{P = L + 1.8W = 5 + 3.6 = 8.6\ ft}$ From Figure 4.2-9, $\mathbf{Q_i = 5\ cfs}$

At d = 0.4 ft, the depressed curb-opening inlet has about 30% more capacity than an inlet without depression.

4.2.7 Combination Inlets

4.2.7.1 Combination Inlets on Grade

On a continuous grade, the capacity of an unclogged combination inlet with the curb opening located adjacent to the grate is approximately equal to the capacity of the grate inlet alone. Thus capacity is computed by neglecting the curb opening inlet and the design procedures should be followed based on the use of Figures 4.2-4, 4.2-5 and 4.2-6.

4.2.7.2 Combination Inlets in Sump

All debris carried by stormwater runoff that is not intercepted by upstream inlets will be concentrated at the inlet located at the low point, or sump. Because this will increase the probability of clogging for grated inlets, it is generally appropriate to estimate the capacity of a combination inlet at a sump by neglecting the grate inlet capacity. Assuming complete clogging of the grate, Figures 4.2-9, 4.2-10, and 4.2-11 for curb-opening inlets should be used for design.

4.2.8 Storm Drain Pipe Systems

4.2.8.1 Introduction

Storm drain pipe systems, also known as *storm sewers*, are pipe conveyances used in the minor stormwater drainage system for transporting runoff from roadway and other inlets to outfalls at structural stormwater controls and receiving waters. Pipe drain systems are suitable mainly for medium to high-density residential and commercial/industrial development where the use of natural drainageways and/or vegetated open channels is not feasible.

4.2.8.2 General Design Procedure

The design of storm drain systems generally follows these steps:

Step 1: Determine inlet location and spacing as outlined earlier in this section.

Step 2: Prepare a tentative plan layout of the storm sewer drainage system including:

- **(a)** Location of storm drains
- **(b)** Direction of flow
- **(c)** Location of manholes
- **(d)** Location of existing facilities such as water, gas, or underground cables

Step 3: Determine drainage areas and compute runoff using the Rational Method

Step 4: After the tentative locations of inlets, drain pipes, and outfalls (including tailwaters) have been determined and the inlets sized, compute of the rate of discharge to be carried by each storm drain pipe and determine the size and gradient of pipe required to care for this discharge. This is done by proceeding in steps from upstream of a line to downstream to the point at which the line connects with other lines or the outfall, whichever is applicable. The discharge for a run is calculated, the pipe serving that discharge is sized, and the process is repeated for the next run downstream. The storm drain system design computation form

(Figure 4.2-12) can be used to summarize hydrologic, hydraulic and design computations.

Step 5: Examine assumptions to determine if any adjustments are needed to the final design.

It should be recognized that the rate of discharge to be carried by any particular section of storm drain pipe is not necessarily the sum of the inlet design discharge rates of all inlets above that section of pipe, but as a general rule is somewhat less than this total. It is useful to understand that the time of concentration is most influential and as the time of concentration grows larger, the proper rainfall intensity to be used in the design grows smaller.

4.2.8.3 Design Criteria

Storm drain pipe systems should conform to the following criteria:

- For ordinary conditions, storm drain pipes should be sized on the assumption that they will flow full or practically full under the design discharge but will not be placed under pressure head. The Manning Formula is recommended for capacity calculations.
- The maximum hydraulic gradient shall not produce a velocity that exceeds 15 ft/s.
- The minimum physical slope shall be 1.0% or the slope that will produce a velocity of 3.0 feet per second when the storm sewer is flowing full, whichever is greater.
- If the potential water surface elevation exceeds 1 foot below ground elevation for the design flow, 1 foot above the top of the pipe, or 1 foot below the gutter flow line, whichever is lowest, adjustments are needed in the system to reduce the elevation of the hydraulic grade line. Surcharge conditions shall be minimized within the storm system.

- Minimum pipe size shall be eighteen (18) inches.
- Storm lines shall extend to within 25 feet of the rear property line, where topography permits or, unless the pipe would extend into the floodplain, wetlands or stream buffer.
- Headwalls are required for pipes that are 42 inches or larger and must meet GA DOT Construction Standards 1125.
- No pipe will be allowed to discharge into a smaller diameter pipe, even though the capacity of the smaller pipe may be greater due to a steeper slope.
- The maximum distance between storm structures shall be 400 feet.
- Step placement in storm structures shall be twelve (12) inches from the bottom and twelve (12) inches separation to the top of the box.
- Top shall be aligned with steps for access to storm structures.
- All storm structures shall have poured inverts.
- HDPE Pipe can be used only outside of the Right of Way and must be installed per the manufacturer's specifications.

STORM DRAIN COMPUTATION SHEET

Route ____________

Computed ____________ Date ____________ Section ____________

Checked ____________ Date ____________ County ____________

Station		Length	Drainage Area "A" Acres		Runoff Coefficient "C"	"A" • "C"		Flow Time Minimum		Rainfall Intensity "I" in / hour	Total Runoff CIA = Q (cfs)	Diameter Pipe inches	Capacity Full cfs	Velocity (fps)		Invert Elev		Manhole Invert Drop	Slope of Drain (ft/ft)
From	To	Feet	Increment	Total		Increment	Total	To Upper End	In Section					Flowing Full	Design Flow	Upper End	Lower End		

Figure 4.2-12 Storm Drain System Computation Form
(Source: AASHTO Model Drainage Manual, 1991)

4.2.8.4 Capacity Calculations

Formulas for Gravity and Pressure Flow

The most widely used formula for determining the hydraulic capacity of storm drain pipes for gravity and pressure flows is the Manning's Formula, expressed by the following equation:

$$V = [1.486\ R^{2/3} S^{1/2}] / n \qquad (4.2.13)$$

Where: V = mean velocity of flow, ft/s
R = the hydraulic radius, ft - defined as the area of flow divided by the wetted flow surface or wetted perimeter (A/WP)
S = the slope of hydraulic grade line, ft/ft
n = Manning's roughness coefficient

In terms of discharge, the above formula becomes:

$$Q = [1.486\ AR^{2/3} S^{1/2}] / n \qquad (4.2.14)$$

Where: Q = rate of flow, cfs
A = cross sectional area of flow, ft^2

For pipes flowing full, the above equations become:

$$V = [0.590\ D^{2/3} S^{1/2}] / n \qquad (4.2.15)$$

$$Q = [0.463\ D^{8/3} S^{1/2}] / n \qquad (4.2.16)$$

Where: D = diameter of pipe, ft

The Manning's equation can be written to determine friction losses for storm drain pipes as:

$$H_f = [2.87\ n^2 V^2 L] / [S^{4/3}] \qquad (4.2.17)$$

$$H_f = [29\ n^2 V^2 L] / [(R^{4/3})(2g)] \qquad (4.2.18)$$

Where: H_f = total head loss due to friction, ft
n = Manning's roughness coefficient
D = diameter of pipe, ft
L = length of pipe, ft
V = mean velocity, ft/s
R = hydraulic radius, ft
g = acceleration of gravity = 32.2 ft/sec2

4.2.8.5 Nomographs and Table

The nomograph solution of Manning's formula for full flow in circular storm drain pipes is shown in Figures 4.2-13, 4.2-14, and 4.2-15. Figure 4.2-16 has been provided to solve the Manning's equation for partially full flow in storm drains.

4.2.8.6 Hydraulic Grade Lines

All head losses in a storm sewer system are considered in computing the hydraulic grade line to determine the water surface elevations, under design conditions in the various inlets, catch basins, manholes, junction boxes, etc.

Hydraulic control is a set water surface elevation from which the hydraulic calculations are begun. All hydraulic controls along the alignment are established. If the control is at a main line upstream inlet (inlet control), the hydraulic grade line is the water surface elevation minus the entrance loss minus the difference in velocity head. If the control is at the outlet, the water surface is the outlet pipe hydraulic grade line.

Design Procedure - Outlet Control

The head losses are calculated beginning from the control point upstream to the first junction and the procedure is repeated for the next junction. The computation for an outlet control may be tabulated on Figure 4.2-17 using the following procedure:

Step 1: Enter in Column 1 the station for the junction immediately upstream of the outflow pipe. Hydraulic grade line computations begin at the outfall and are worked upstream taking each junction into consideration.

Step 2: Enter in Column 2 the outlet water surface elevation if the outlet will be submerged during the design storm or 0.8 diameter plus invert elevation of the outflow pipe, whichever is greater.

Step 3: Enter in Column 3 the diameter (D_o) of the outflow pipe.

Step 4: Enter in Column 4 the design discharge (Q_o) for the outflow pipe.

Step 5: Enter in Column 5 the length (L_o) of the outflow pipe.

Step 6: Enter in Column 6 the friction slope (S_f) in ft/ft of the outflow pipe. This can be determined by using the following formula:

$$\mathbf{S_f = (Q^2)/K} \qquad \mathbf{(4.2.19)}$$

Where: S_f = friction slope

K = $[1.486\ AR^{2/3}]/n$

Step 7: Multiply the friction slope (S_f) in Column 6 by the length (L_o) in Column 5 and enter the friction loss (H_f) in Column 7. On curved alignments, calculate curve losses by using the formula $H_c = 0.002\ (\Delta)(V_o{}^2/2g)$, where Δ = angle of curvature in degrees and add to the friction loss.

Step 8: Enter in Column 8 the velocity of the flow (V_o) of the outflow pipe.

Step 9: Enter in Column 9 the contraction loss (H_o) by using the formula:

$$\mathbf{H_o = (0.25\ V_o{}^2)/2g}\text{, where } g = 32.2\ \text{ft/s}^2$$

Step 10: Enter in Column 10 the design discharge (Q_i) for each pipe flowing into the junction. Neglect lateral pipes with inflows of less than 10% of the mainline outflow. Inflow must be adjusted to the mainline outflow duration time before a comparison is made.

Step 11: Enter in Column 11 the velocity of flow (V_i) for each pipe flowing into the junction (for exception see Step 10).

Step 12: Enter in Column 12 the product of Q_i x V_i for each inflowing pipe. When several pipes inflow into a junction, the line producing the greatest Q_i x V_i product is the one that should be used for expansion loss calculations.

Step 13: Enter in Column 13 the controlling expansion loss (H_i) using the formula:

$$\mathbf{H_i = (0.35\ (V_1{}^2))/2g}$$

Step 14: Enter in Column 14 the angle of skew of each inflowing pipe to the outflow pipe (for exception, see Step 10).

Step 15: Enter in Column 15 the greatest bend loss (H) calculated by using the formula $\mathbf{H = [KV_i{}^2]2g}$ where K = the bend loss coefficient corresponding to the various angles of skew of the inflowing pipes.

Step 16: Enter in Column 16 the total head loss (H_t) by summing the values in Column 9 (H_o), Column 13 (H_i), and Column 15 (H_Δ).

Step 17: If the junction incorporates adjusted surface inflow of 10% or more of the mainline outflow, i.e., drop inlet, increase Ht by 30% and enter the adjusted H_t in Column 17.

Step 18: If the junction incorporates full diameter inlet shaping, such as standard manholes, reduce the value of H_t by 50% and enter the adjusted value in Column 18.

Step 19: Enter in Column 19 the FINAL H, the sum of H_f and H_t, where H_t is the final adjusted value of the H_t.

Step 20: Enter in Column 20 the sum of the elevation in Column 2 and the Final H in Column 19. This elevation is the potential water surface elevation for the junction under design conditions.

Step 21: Enter in Column 21 the rim elevation or the gutter flow line, whichever is lowest, of the junction under consideration in Column 20. If the potential water surface elevation exceeds 1 foot below ground elevation for the design flow, the top of the pipe or the gutter flow line, whichever is lowest, adjustments are needed in the system to reduce the elevation of the Hydraulic Grade Line (H.G.L.).

Step 22: Repeat the procedure starting with Step 1 for the next junction upstream.

Step 23: At last upstream entrance, add $\mathbf{V_1^2 / 2g}$ to get upstream water surface elevation.

4.2.8.7 Minimum Grade

All storm drains should be designed such that velocities of flow will not be less than 3.0 feet per second at design flow or lower, with a minimum slope of 1.0%. For very flat flow lines the general practice is to design components so that flow velocities will increase progressively throughout the length of the pipe system. Upper reaches of a storm drain system should have flatter slopes than slopes of lower reaches. Progressively increasing slopes keep solids moving toward the outlet and deter settling of particles due to steadily increasing flow streams.

The minimum slopes are calculated by the modified Manning's formula:

$$\mathbf{S = [(nV)^2]/[2.208\,R^{4/3}]} \qquad \mathbf{(4.2.20)}$$

Where: S = the slope of the hydraulic grade line, ft/ft

n = Manning's roughness coefficient

V = mean velocity of flow, ft/s

R = hydraulic radius, ft (area dived by wetted perimeter)

4.2.8.8 Storm Drain Storage

If downstream drainage facilities are undersized for the design flow, a structural stormwater control may be needed to reduce the possibility of flooding. The required storage volume can also be provided by using larger than needed storm drain pipe sizes and restrictors to control the release rates at manholes and/or junction boxes in the storm drain system. The same design criteria for sizing structural control storage facilities are used to determine the storage volume required in the system (see Section 2.2 for more information).

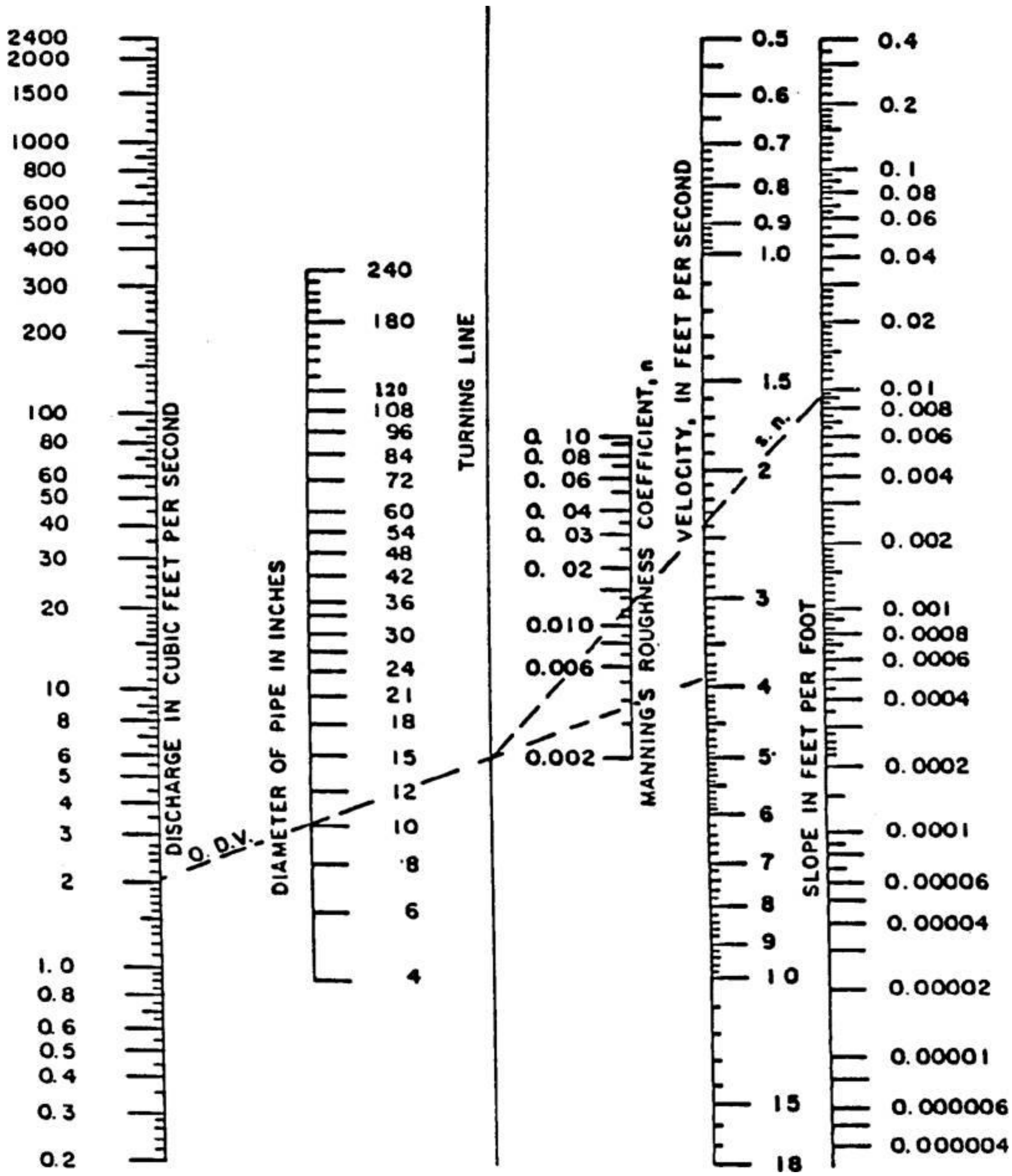


Figure 4.2-13 Nomograph for Solution of Manning's Formula for Flow in Storm Sewers

(Source: AASHTO Model Drainage Manual, 1991)

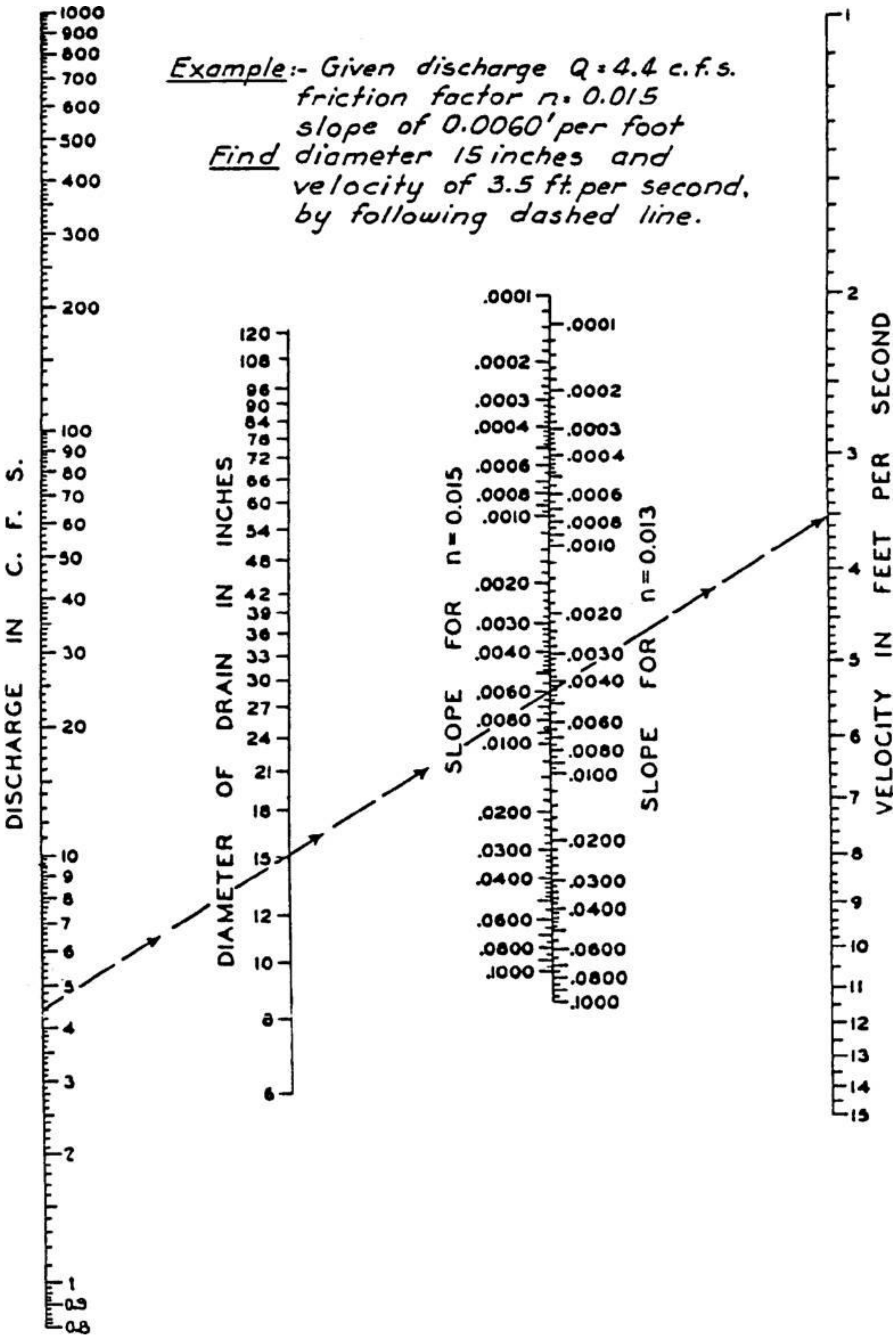


Figure 4.2-14 Nomograph for Computing Required Size of Circular Drain, Flowing Full n=0.013 or 0.015

(Source: AASHTO Model Drainage Manual, 1991)

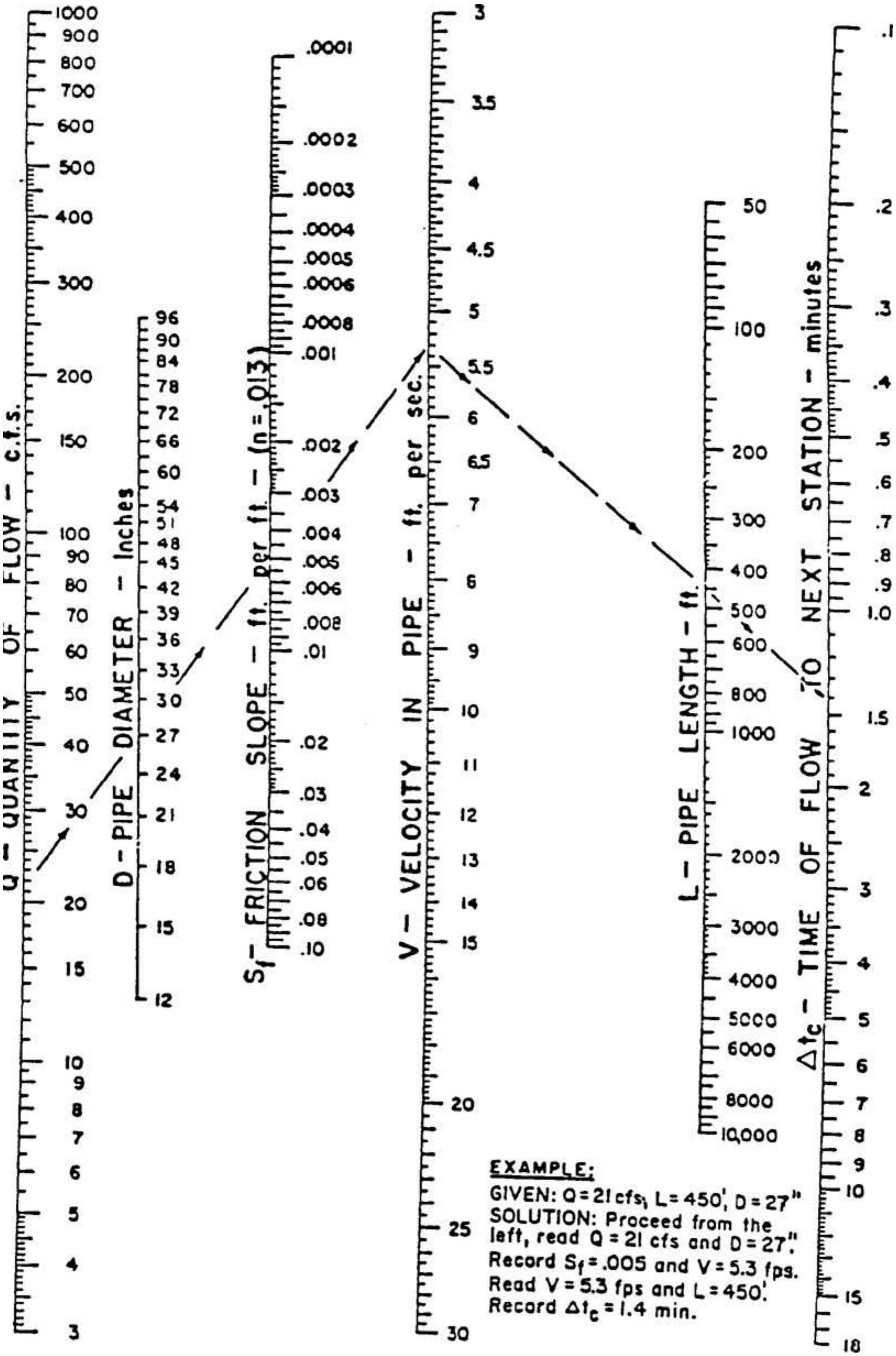
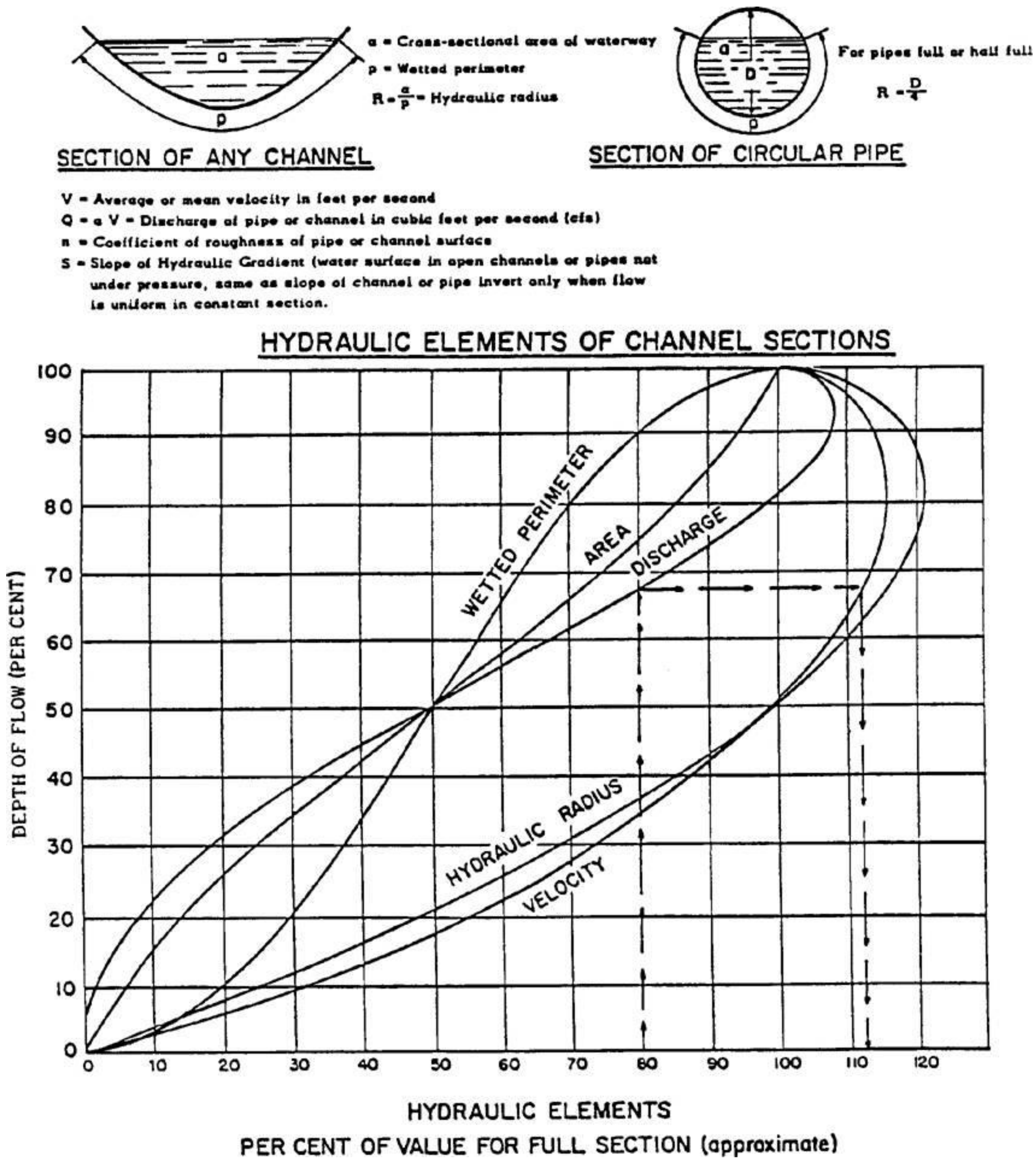


Figure 4.2-15 Concrete Pipe Flow Nomograph

(Source: AASHTO Model Drainage Manual, 1991)



V = Average of mean velocity in feet per second

Q = Discharge of pipe or channel in cubic feet per second

S = Slope of hydraulic grade line

Figure 4.2-16 Values of Various Elements of Circular Section for Various Depths of Flow

(Source: AASHTO Model Drainage Manual, 1991)

Hydraulic Grade Line

Project ____________________

Station	Outlet Water Surface Elev.	D_o	Q_o	L_o	Sf_o %	H_f	Junction Loss												Final H	Inlet Water Surface Elev.	Rim Elev.
							V_o	H_o	Q_i	V_i	Q_iV_i	$\frac{V_i^2}{2g}$	H_i	Angle	H_Δ	H_t	1.3 H_t	0.5 H_t			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)		(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)

$$H_f = 0.35\frac{V_i^2}{2g} \qquad H_o = 0.25\frac{V_o^2}{2g} \qquad H_\Delta = K\frac{V_i^2}{2g}$$

Final $H = H_f + H_i$

$$H_t = H_o + H_i + H_\Delta$$

20° K =	0.70	20° K =	0.47	20° K =	0.16
20° K =	0.66	20° K =	0.38	15° K =	0.10
20° K =	0.61	20° K =	0.28		
20° K =	0.55	20° K =	0.22		

Figure 4.2-17 Hydraulic Grade Line Computation Form
(Source: AASHTO Model Drainage Manual, 1991)

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